Experimental Investigation of Efficiency of Tuned Single and Double Mass Damper and its Application in the Form of an Additional Upper Floor for Seismic Protection of Existing Multistory Buildings

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Abstract

There are a rather large number of studies on theoretical investigation of behavior of buildings with tuned mass dampers under various impacts. However, the experimental studies in this area are quite limited. This work to a certain extent fills the gap in experimental studies aimed at investigation of behavior of buildings with tuned single and double mass dampers. Based on these investigations different types of dampers were designed and implemented for seismic protection of existing buildings. Linear and non-linear analyses and dynamic tests of these buildings were carried out confirming the efficiency of tuned mass dampers.

Keywords

Single Mass Damper; Double Mass Damper; Model Investigation; Analysis; Application; Existing Building; Dynamic Test; Efficiency; Seismic Protection

Introduction

In some of his papers the author of this article has described two works implemented through vibration tests on the Series 111 nine-storey full-scale residential buildings equipped with tuned mass dampers in the form of flexible upper floor (FUF) and isolated upper floor (IUF) in Vanadzor city (formerly Kirovakan), Armenia. These are low-cost seismic protection technologies which were implemented for the first time in the world without interruption of the use of the buildings. Application of such pioneering technologies became possible due to below described experimental and theoretical investigations.

Experimental Investigation on Large-Scale Model of the Behavior of Building Equipped with Different Types of Tuned Mass Dampers

The current studies have been conducted on a model of the same Series 111 nine-storey frame building. The model of reinforced concrete on a scale of 1:5 has been designed and made using the principle of simple similarity. It had nine columns forming two 120 cm spans in mutually perpendicular directions, with axial dimensions of 240x240 cm. In one of the directions, the spatial stiffness of the model was provided by three frames with strong bearing beams (frame design), and in the other – by three frames with weak binding beams and a single shear wall located in one of the spans in the plane of the middle frame (braced-frame design). The cross-sections of columns were 8x8 cm, bearing beams – 8x10.4(h) cm and binding beams – 8x5(h) cm. Floors consisted of prefab hollow-core model slabs with thickness of 4.4cm, whereas the prefab panels of the shear wall were 2.8 cm thickness. In accordance with the structural concept of the Series 111 buildings, the shear wall panels were connected to columns by welding the embedded items, and their connection to the beams was provided by casting concrete over the dowels protruding from the shear wall prefab panels. The total height of the model was 5.9 m, with the height of each storey being 0.6 m; foundation beams were 0.4 m height and the column steel caps extended beyond the 9th storey slab by 0.1 m.

General view of the model before placement of tuned mass dampers is shown in Figure 1, in which cast iron weights were suspended from the model's slabs to provide vertical loads for the creation of the necessary level of stress-strain state in the structural elements of

the model. More detailed information on the design, construction and testing of this model without tuned mass dampers can be found in other publications of the author and is excluded from provision here for brevity.
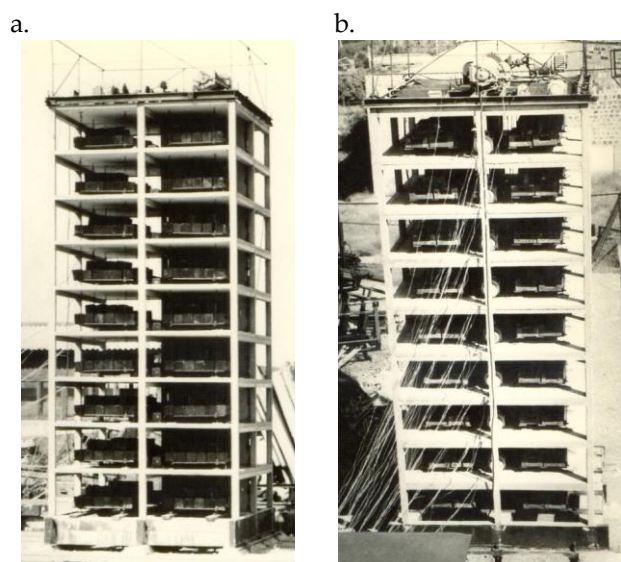

FIG. 1 SERIES 111 NINE-STOREY FRAME BUILDING MODEL DESIGNED AND CONSTRUCTED ON A SCALE OF 1:5;
A – FRONT VIEW, B – SIDE VIEW

Before the tuned mass dampers have been placed, the model vibrations were induced in both directions by laboratory vibration machines specially developed and made for model tests. The vibration machines were placed on the 9th storey's slab and rigidly connected to a steel frame, which in turn was welded to the steel caps of all nine columns of the model (Fig. 2).
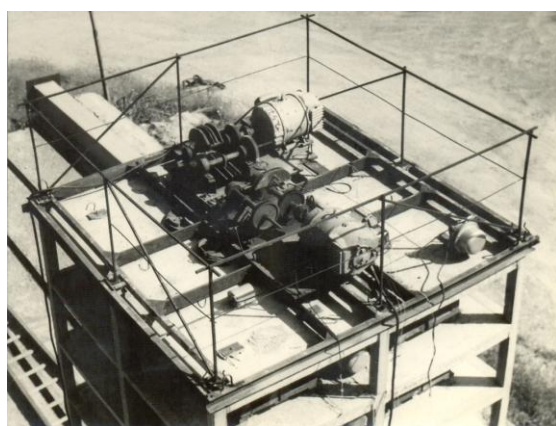

FIG. 2 LABORATORY VIBRATION MACHINES FOR TESTING LARGE MODELS, PLACED ON THE SLAB OF THE 9TH STOREY (BEFORE MOUNTING THE DAMPERS) WITH A STEEL FRAME RIGIDLY FIXED TO THE STEEL CAPS OF THE NINE-STOREY FRAME BUILDING MODEL COLUMNS

The vibration machines were mounted in such a manner that the exciting forces created during rotation of the weights which were eccentrically placed on the

vibrators shafts acted in the vertical planes passing in mutually perpendicular directions through the middle axes (frames) of the model. The revolutions of the vibration machines' DC motors were continuously variable with the help of a special device, thus providing harmonic vibrations in the range from 0 to 10 Hz. The parameters of the forced vibrations were measured by large displacement seismographs, as well as accelerographs placed at the level of all floors along the central vertical axis of the model, with exception of the devices on the 9th floor, where they were placed outside the central zone of the model's plan, as it was occupied by the vibration machines.

Vibration tests of the Series 111 nine-storey full-scale frame buildings equipped with tuned mass dampers in the form of FUF or IUF have been mentioned hereinabove. Those systems were single mass dampers. Conversely, the investigation subject of this experimental work was tuned double mass damper. Prior to mounting such damper as an additional tenth floor, the dynamic characteristics of the model (periods and damping ratios) have been determined. The vibration machine was used to induce forced resonant vibrations of 1st, 2nd and 3rd modes. The values of periods and damping ratios calculated based on the records of these vibrations are indicated in Table 1.

TABLE 1 PERIODS OF OSCILLATIONS AND DAMPING RATIOS OF THE NINE-STOREY FRAME BUILDING MODEL AT THE STAGE OF ELASTIC BEHAVIOR UNDER THE FIRST THREE MODES OF RESONANT VIBRATIONS, PRIOR TO MOUNTING THE TUNED DOUBLE MASS DAMPER

Direction of the tests	Periods of oscillations (sec) for the			Damping ratio ξ, %
	1st mode T_1	2nd mode T_2	3rd mode T_3	
Along the frames with strong bearing beams (frame design)	0.326	0.133	0.075	4.3
Along the frames with weak binding beams (braced-frame design)	0.280	0.088	0.049	4.0

Experimental studies of the nine-storey frame building model with tuned double mass damper were intended to carry out under its loading only in the direction of the frames with weak binding beams and shear wall located in one of the spans in the plane of the middle frame. Given this circumstance, one of the vibration machines was dismantled. In order to reach the cracking stage, in the mentioned direction the model

was subjected to several phases of dynamic loading with gradual increase of the mass of the off-center weights on the vibrator shafts. In these conditions the shear wall panels suffered light damage in the form of inclined, intersecting thin cracks, whereas the fundamental period of oscillations increasing by 1.25 times compared to the initial one and was equal to 0.35 sec (ω_I=2.86 Hz). Almost no change of the damping ratio was observed (ξ=4%), while the 9th storey's floor displacement amplitude was A=3.1 cm.

The obtained values served as a basis to choose the mass and stiffness parameters of the first damper, which was named as "the main". As it turned out that the necessary horizontal stiffness of the mentioned additional tenth storey, i.e. the tuned mass damper, could be achieved by using square iron posts with cross-section of 14x14 mm, welded to the sides of the steel caps of the nine-storey frame building model's columns. The stiffness of the model main damper ceiling could be ensured by means of 45x45x4 mm L-shaped rolled steel profiles, whereas the mass could be accumulated by R/C plates placed on these profiles (Fig. 3). Thus, the design of the main damper created on the nine-storey model is similar to that of the FUF, the design of which has been developed by the author in this article, and later implemented and tested with his direct participation.
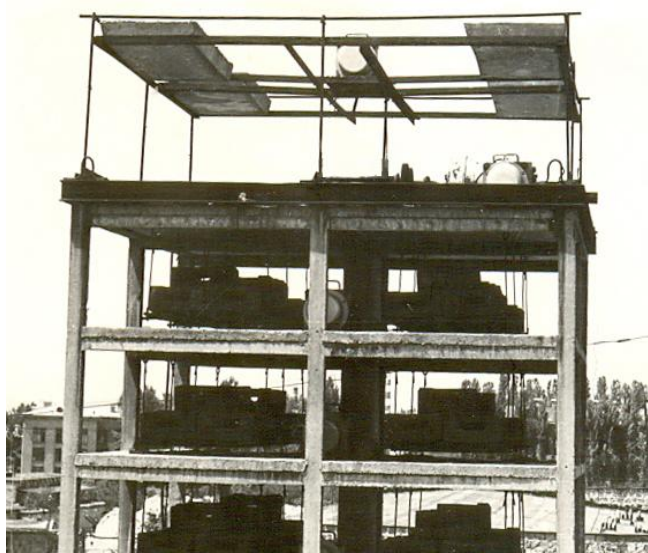

FIG. 3 FRAGMENT OF THE NINE-STOREY MODEL'S UPPER PART WITH A VIEW OF THE ADDITIONAL TENTH STOREY, I.E. THE MAIN DAMPER TOGETHER WITH PLACED R/C PLATES THAT CONSTITUTE ITS MASS

The studies were aimed at determining the efficiency and damping properties of the tuned double mass damper and comparing them to those of the single mass damper. The question of optimal damping in the

main damper was not dealt with. The main damper's tuning f^2_1=f^2_{MD}÷ω^2_1 (without consideration of damping) was performed immediately on the damper. Initially, the main damper was tuned to the f_{MD} frequency, very close to the resonance frequency ω_1 of the model's 1st mode oscillations. Afterwards, using incremental change of the damper's mass, the optimal tuning was determined at which the maximum efficiency of damping was achieved. It turned out that the optimal tuning requires the main damper's mass accumulation up to the value of m_{MD}=360 kg, which is half of the model's single storey weight (without kentledge) m_{storey}=720 kg. Hence, given that total mass of the model's nine storeys is M_M=720x9=6480 kg, then the relative mass is v=m_{MD}÷M_M=360÷6480=0.056 (or 5.6%), and the tuning f^2_1=1.0.

The approach that takes into account the damping in optimization of parameters for tuned mass dampers differs from the case for dampers without damping, since optimal values of tuning and relative viscous (or inelastic) resistance for a given value of v are the ones to be determined. Optimal values of tuning for hypotheses of viscous resistance or internal inelastic resistance are determined in the same way. In our case the optimal tuning with consideration of damping differing from the f^2_1 value is determined by the following formula:

$$f_{op}^2 = \frac{1}{1+\upsilon} = \frac{1}{1.056} = 0.95$$

Once the main damper was tuned, it was subjected to dynamic loading with variable frequency. Figure 4a shows fragments of vibration records obtained by seismographic instrumentation at the slab of the model's ninth storey and at the level of main damper's top. It was derived from the obtained records that the maximum oscillation amplitude of the model's upper part with the main damper is 1.05 cm, which is almost 3 times less compared to the oscillation amplitude of the model's upper part without a damper (3.1cm). The maximum oscillation amplitudes of the model's upper part with the damper were approximately the same before and after passing through resonance and corresponded to frequencies of 2.5 Hz and 3.3 Hz which are almost symmetrical in relation to the resonant frequency of the model without a damper (2.86 Hz). It has to be noted that before passing through resonance the damper acted in the same phase with the model, whereas after passing through resonance it acted in anti-phase relative to the model. The main damper oscillation amplitude was about 3-4

times larger than the amplitudes of the model's upper part oscillation. The free oscillations of the optimally tuned main damper have also been recorded (Fig. 4b). The damping ratio of the main damper determined from this record was ξ=0.7%.
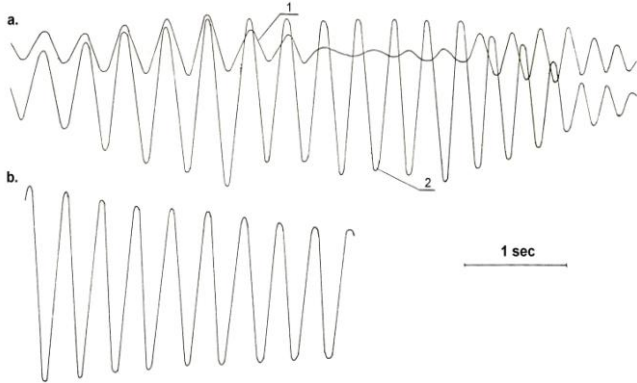

FIG. 4 FRAGMENTS OF VIBRATION RECORDS AT THE TOP OF THE NINE-STOREY MODEL WITH A SINGLE MASS DAMPER (1) AND AT THE LEVEL OF THE DAMPER'S TOP (2) UNDER DYNAMIC LOADING CREATED BY VIBRATION MACHINE (A), AS WELL AS FRAGMENT OF THE SINGLE MASS DAMPER'S NORMAL MODE OSCILLATIONS RECORD (B)

Upon completion of the model tested with the main damper, a second weight was added to its ceiling and hence, the single mass damper was turned into a double mass one. The second mass of the damper could be accumulated by weights placed in a 30x40x10 cm metallic box weighing 10 kg. The metallic box was suspended from the main damper's ceiling by round steel rods with a diameter of 8 mm and length of 30 cm. The general view of the second suspended mass is shown in Figure 5. The tuning, or in other words, selection of the optimal value for the second mass was performed in the same manner as the tuning of the main damper. At the optimal tuning, the weight for the second mass of the damper turned out to be 40 kg, and the partial frequency of its normal mode was 2.65 Hz. The relative masses and tunings of the double mass damper were, respectively:

$$\upsilon_d = \frac{m_{d1} + m_{d2}}{M_M} = \frac{360 + 40}{9 \times 720} = 0.0617;$$

$$f_1^2 = 1.0; \quad f_2^2 = 0.93; \quad \upsilon_2 = \frac{m_{d2}}{m_{d1}} = \frac{40}{360} = 0.11$$

The total mass of the double mass damper turned to be 11% higher than that of the single mass damper. Over the course of tuning the created damper system, control records of model oscillations with double mass damper were obtained, as well under harmonic impact with variable frequency. The records of these oscillations provided in Figure 6 along with the ones

shown in Figure 4 were used to derive the amplitude-frequency characteristics of the model (Fig. 7).
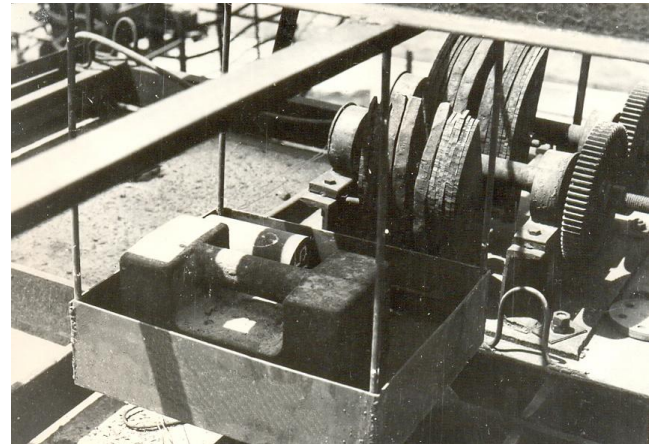

FIG. 5 FRAGMENT OF THE NINE-STOREY MODEL'S UPPER PART WITH THE VIEW OF THE SECOND MASS CREATED BY WEIGHTS PLACED IN A METALLIC BOX AND SUSPENDED FROM THE MAIN DAMPER
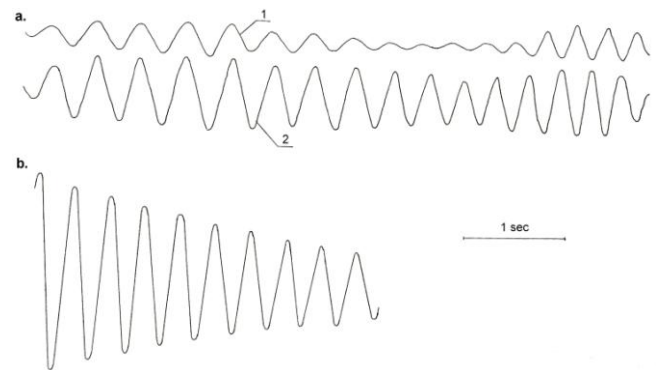

FIG. 6 FRAGMENTS OF VIBRATION RECORDS AT THE TOP OF THE NINE-STOREY MODEL WITH A DOUBLE MASS DAMPER (1) AND AT THE LEVEL OF THE DAMPER'S TOP (2) UNDER DYNAMIC LOADING CREATED BY VIBRATION MACHINE (A), AS WELL AS FRAGMENT OF THE DOUBLE MASS DAMPER'S NORMAL MODE OSCILLATIONS RECORD (B)

The obtained results indicate that the tuned double mass damper is more efficient than the single one, since the damping coefficient for the double mass damper reaches 4.4, which is 46.7% higher than that of the single mass damper. Moreover, the range of the frequencies damped is about 50% wider. These results actually somewhat exceeded the expectations. Comparing Figures 4 and 6 one may notice that the oscillation amplitudes of the main damper have significantly decreased. Before adding the second mass the maximum amplitude of the tuned single mass damper was around 3.5 cm, whereas after adding the second mass, it amounted to 1.4 cm. Normal mode oscillation records showed that its damping ratio is ξ=1.75%, i.e. 2.5 larger compared to that of the single mass damper. Apparently, increased damping effect of the tuned double mass damper is explained by mutual influence of the main damper

and the second mass suspended from it, whereby the energy of oscillations is re-distributed. Thus, it was found that under harmonic oscillations induced by a vibration machine, the tuned double mass damper had a higher efficiency than the single one.
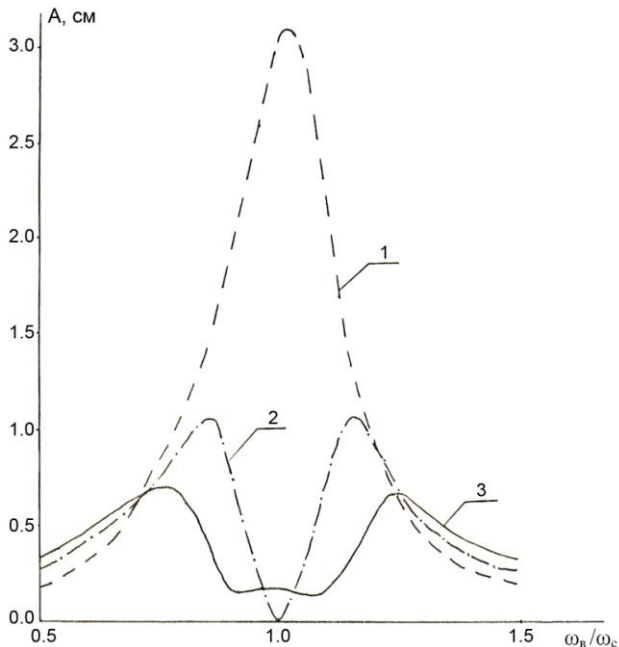

FIG. 7 AMPLITUDE-FREQUENCY CHARACTERISTICS OF THE NINE-STOREY MODEL: (1) WITHOUT DAMPER; (2) WITH THE SINGLE MASS DAMPER; AND (3) WITH THE DOUBLE MASS DAMPER

Based on the summary of the above, it can be stated that the conducted experimental studies confirm the rather high efficiency of tuned mass dampers and that they are undoubtedly worth being used to increase the seismic resistance of buildings and structures. The question of the utilization of one or another type of damper depending on the number of masses (single, double or multi-mass) or on structural scheme (FUF or IUF, etc.) at the top of actual buildings is to be addressed considering many factors including the number of storeys, structural concept and technical condition of the buildings, circumstances that limit the damper movement, necessity to utilize the space inside the damper, cost effectiveness of the damper and many other factors.

Linear Analyses of a Building with and without TMD in the Form of an Additional Upper Floor

Thus, basically TMD is a single-degree-of-freedom appendage of the primary structure. Dampers have been widely investigated in connection with seismic protection problems. The natural frequency of TMD should be equal to the forced vibration frequency of the structure to be protected, which as a rule is represented in a form of a single-degree-of-freedom (SDOF) system. However, during earthquakes, forced vibrations neither are harmonic, nor have a preset frequency and buildings are not SDOF systems. But in spite of the chaotic nature of the ground motion, response of linear oscillator is similar to harmonic vibration process with the period equal to that of linear oscillator. Therefore, if the first vibration mode is the most significant one during earthquakes, then the natural frequency of the damper should be equal to the first mode frequency of structure vibration. An additional upper floor for the buildings has been proposed as a vibration damper – TMD and it could be erected on the existing buildings to increase their seismic resistance, without empty the building.

The attempt to find the optimal parameters of TMD in the form of an additional flexible upper tenth floor (AFUF) in 9-story frame buildings, using accelerograms of various earthquakes is presented below. At this step the building design model including the TMD is assumed to be a cantilever beam with masses concentrated at the floor levels. The equations of the forced vibrations of such a system are given by the formula:

$$m_k(\ddot{y}_k + \ddot{y}_0) + a_k(y_k - y_{k-1}) + \mu_k a_k(\dot{y}_k - \dot{y}_{k-1}) - a_{k+1}(y_{k+1} - y_k) - \mu_{k+1} a_{k+1}(\dot{y}_{k+1} - \dot{y}_k) = 0,$$

where m_k, a_k, y_k are the mass, stiffness and displacement of the kth floor of the building, $m_r = m_{10}$, $a_r = a_{10}$, $y_r = y_{10}$ are the mass, stiffness and displacement of the TMD-AFUF, $\mu_k = \alpha / \omega$ is the coefficient of viscous damping of the kth floor, and $\ddot{y}_0(t)$ is the ground acceleration (accelerogram).

The values of floors' stiffness and mass of the investigated building are as follows: $a_1 = a_2 = \ldots = a_9 = 897000$ kN/m; $m_1 = m_2 = \ldots = m_8 = 360$ kNxs^2/m; $m_9 = 430$ kNxs^2/m. Substituting these data (at $\mu_k = 0$), the periods of the first three vibration modes of the building in the direction of shear walls (with door opening at column) without TMD were obtained: $T_1 = 0.778$ s, $T_2 = 0.261$ s, $T_3 = 0.159$ s. The building with TMD was analyzed using 12 accelerograms (Tab. 2) of strong earthquakes with the purpose to obtain the minimal values of the base shear forces, to determine corresponding optimal values of $v = mr/m1$ and $d = ar/a1$, and to compare the received results with those calculated for the building without TMD.

TABLE 2 OPTIMAL PARAMETERS OF TMD AND BASE SHEAR FORCES OF A 9-STORY BUILDING ANALYZED BY 12 TIME HISTORIES WITH AND WITHOUT DAMPER

Earthquakes	Optimal parameters determined for each time history		Base shear forces (kN) of the building	
	v	d	with TMD	without TMD
Ferndale, USA 7.10.1951, 44W	0.50	0.0150	1600	2680
Ferndale, USA 7.10.1951, 46E	1.25	0.0334	2740	4320
Ferndale, USA 21.12.1954, 44W	0.50	0.0100	7380	11220
Ferndale, USA 21.12.1954, 46E	1.00	0.0075	8600	12560
Ulcinj-2, Yugoslavia 15.04.1979, N-S	1.00	0.0075	3900	5180
Ulcinj-2, Yugoslavia 15.04.1979, N-E	1.25	0.0334	7260	11700
Herceg Novi, Yugoslavia 15.04.1979, N-S	1.00	0.0265	6200	10080
Herceg Novi, Yugoslavia 15.04.1979, N-E	1.25	0.0334	5200	7780
Ferndale, USA 3.10.1941, H60	0.75	0.0334	1318	1856
Hollister, USA 9.03.1949, H21	0.50	0.0100	2240	3900
Eureka, USA 21.12.1954, H10	0.50	0.0200	5620	8400
Taft, USA 12.01.1954, H70	0.50	0.0120	1100	2040

The results show that AFUF reduces the lateral forces by about 35% in average. Seismic loads and lateral forces, as well as displacements along the height of the building for both cases with and without TMD are shown in Figure 8. These results also indicate that the efficiency of a single mass damper in the form AFUF tuned to the first mode of building vibration is not so high. The mean values of the optimal parameters derived from Table 1 are the following: $v = 0.83$ and $d = 0.02$. Thus, the mass of TMD is equal to about 9% of the total mass of the building and its stiffness is about 50 times less than the stiffness of the building's typical floor.
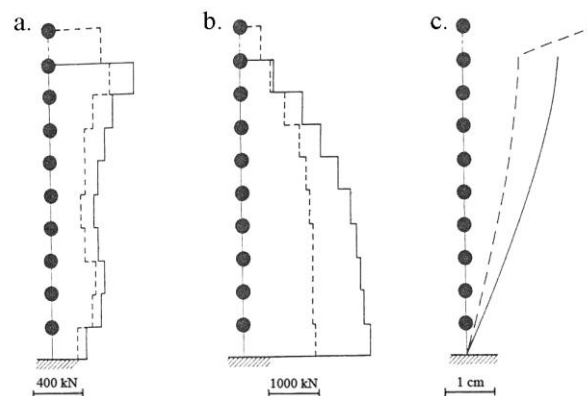

FIG. 8 SEISMIC LOADS (A), LATERAL FORCES (B) AND DISPLACEMENTS (C) OF THE 9-STORY BUILDING WITHOUT (SOLID LINE) AND WITH (DASHED LINE) THE TMD ANALYZED BY THE 9.03.1949, H21 HOLLISTER ACCELEROGRAM

However, three dampers tuned to the first three vibrations modes of the building are considered much more effective and, therefore, a building structural solution with three TMDs has been proposed. When analyzing any building with TMDs, the number of vibration modes that should be taken into account is equal to the number of TMDs, with addition of at least the next three modes. Thus, for the buildings with three dampers as it is schematically illustrated in Figure 9, at least six vibration modes should be encompassed in the analysis. The multi-version analyses of such structure led to the conclusion that in this case optimal stiffness and mass correlations of dampers held the feature of significant reduction of shear forces and displacements (for about 2 times) compared to the building without TMDs.
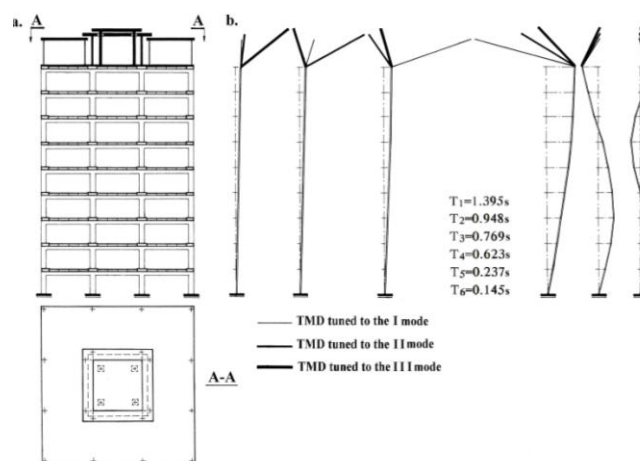

FIG. 9 SCHEMATIC OF THE 9-STORY BUILDING WITH THREE TMDS TUNED TO ITS FIRST THREE NATURAL FREQUENCIES (A) AND SIX VIBRATION MODES TAKEN INTO ACCOUNT (B) IN THE ANALYSIS OF THE BUILDING

It has to be noted that in case of consideration of non-linearity for both the building and TMD structural elements its effectiveness significantly increases. Results of the non-linear analysis are discussed below.

Reduction of lateral forces and displacements in the building with TMD takes place due to increase of vibration period of the whole system (building plus the TMD) and decrease of the first mode participation factors. However, a new type of second vibration mode appears and becomes prevalence, which results in the TMD oscillations in anti-phase relative to the building along the whole duration of the earthquake accelerogram.

Justification of Transition from the Concept of Flexible Upper Floor to the Concept of Isolated Upper Floor

The TMD in the form of AFUF considered above was implemented on the R/C 9-story building (Fig. 10). It can be seen that the AFUF represents a structure made of steel columns supporting a thick R/C slab.

FIG. 10 GENERAL VIEW OF THE 9-STORY BUILDING WITH THE TMD-AFUF CONSTRUCTED ABOVE IT

This building was tested using a powerful vibration machine installed on the slab of the 9th floor. Tests were carried out in two stages before and after erection of AFUF. Comparison of the results of tests with the analytical results has confirmed the fittingness of AFUF for application in the existing building. Nevertheless, it became obvious that such a structural solution of AFUF contains some deficiencies from the practical point of view. In order to rigidly connect steel columns to the structural elements of the building, these columns should have sufficiently big cross-sections. But in this case the only way to provide the needed flexibility to the AFUF is to increase the height of steel columns (more than 4 m). However, this measure on one side reduces the resistance of AFUF against wind and on the other side it raises its gravity center very high above the existing building.

Therefore, during strong ground motions the flexible upper floor, though, protecting the existing building, may itself suffer severe damages or even be destroyed causing damages to the building. Another deficiency is that no exterior and interior walls shall be constructed around and inside the space of the flexible floor as they will restrict its large horizontal displacement. Because of that and the possibility of partial or total destruction of AFUF during strong earthquakes, it cannot be occupied and does not possess sufficient reliability. All the above justifies the necessity to change the conceptual solution of this floor while keeping its idea. It has been suggested that flexibility should be provided to the damper using laminated rubber bearings (LRBs). Obviously, in such case the known AFUF will turn into an additional isolated upper floor – AIUF (Fig. 11).
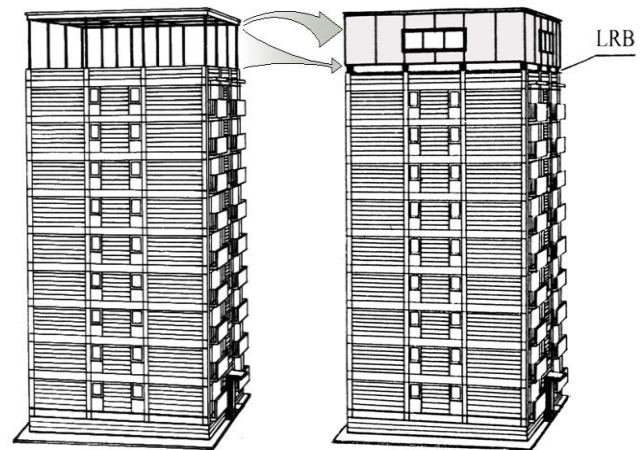

FIG. 11 CHANGE OF THE CONCEPT OF THE ADDITIONAL UPPER FLOOR FROM FLEXIBLE TO ISOLATED

Thus, the thin flexible columns are changed to seismic isolation LRBs and the slab, representing the mass of the flexible floor, is also changed to a whole upper floor connected to the existing building via LRBs. It is important to note that in the proposed solution the R/C slab of AIUF is constructed right above the LRBs and comprises the largest portion of the damper (AIUF) mass. Therefore, the gravity center of the damper in this new structural solution is very close to the existing building. Actually, AIUF itself above the isolation interface is a rigid structure, which being supported by LRBs undergoes practically no deformations during the earthquakes. Consequently, the suggested new concept of a TMD creation on top of the existing building allows not only increasing its seismic resistance and reliability of the whole system, but also enlarging its useful space, which can be used for many different purposes.

Non-Linear Seismic Response Analysis and Dynamic Tests of the 9-Story Full-Scale Existing Building before and after Erection of AIUF

The method of AIUF was used in earthquake protection design and implementation for two existing R/C 9-story standard design buildings (Fig. 12). A special structure connecting the AIUF to the building was developed.

FIG. 12 GENERAL VIEWS OF THE TWO EXISTING R/C 9-STORY APARTMENT BUILDINGS PROTECTED BY AIUF

Free vibration periods of this type of buildings were determined based on the measurements of micro oscillations on a large number of undamaged buildings. The following results were obtained: first mode vibration period in transverse direction (along the R/C frames with weak beams and shear walls) T_{trans}= 0.48 sec in average, and in longitudinal direction (along the R/C frames with strong beams) T_{long}= 0.59 sec in average. Similar results for undamaged buildings are indicated by other authors. The design model of the building is presented in Figure 13.

Seismic response analysis was carried out for the building with and without AIUF, using degrading tri-linear model for columns and bilinear model for rubber bearings, as well as the Melkumyan model for shear walls, and using 7.12.1988, X direction Spitak Earthquake accelerogram scaled to 0.4g. The main results of non-linear seismic response analysis are given in Figure 14 and Table 3. The small scale of Figure 14 makes it hard to see and understand the behavior of LRBs. Therefore, the hysteresis loops for one LRB in a larger scale are presented in Figure 15.
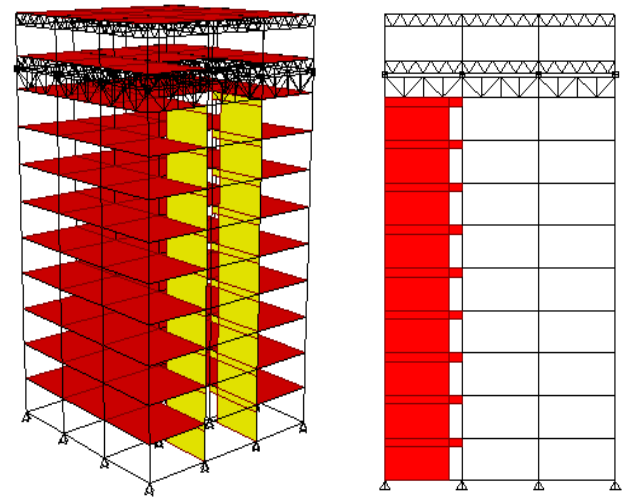

FIG. 13 DESIGN MODEL OF 9-STORY BUILDING PROTECTED BY AIUF

without AIUF   With AIUF   Without   With
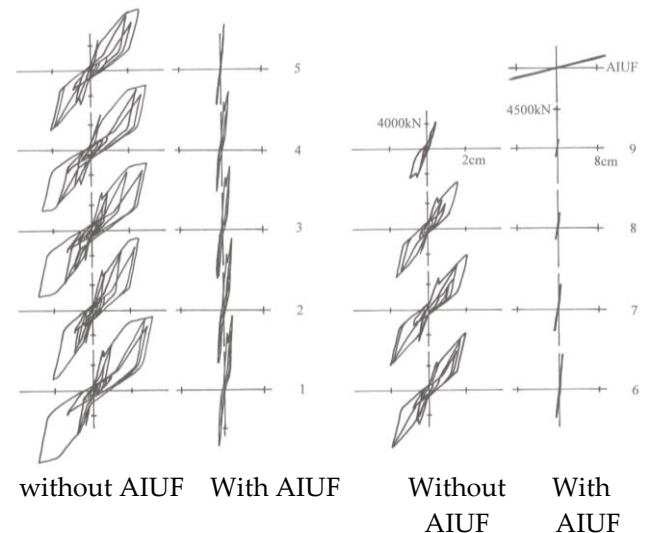   AIUF   AIUF

FIG. 14 RESTORING FORCE - FLOOR DRIFT RELATIONSHIPS FOR EACH FLOOR OF THE BUILDING WITHOUT AND WITH AIUF
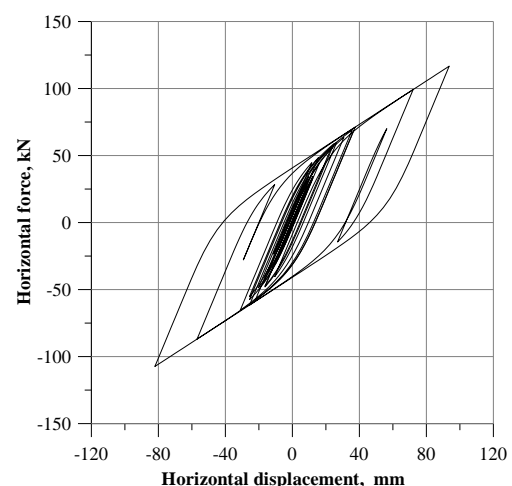

FIGURE. 15 FORCE-DISPLACEMENT RELATIONSHIP FOR A RUBBER BEARING OF AIUF

From the obtained results, it can be seen that the R/C columns and shear walls of the building protected with TND are mainly in the cracking stage, although

yielding does occur in the shear walls of the lower four floors. Comparative analysis of the same building without TMD shows that under the action of the same accelerogram the columns are in the yielding and shear walls are in the ultimate stages of deformation. Thus, the TMD provides sufficient earthquake protection to the building under consideration as the stress-strain state level in bearing structures significantly decreases. The non-linear seismic response analysis proves that with AIUF acting as a TMD seismic loads experienced by the building could be reduced along the height of the building by about 2.5 times in average.

TABLE 3 THE VALUES OF HORIZONTAL SEISMIC LATERAL FORCES AND STAGES OF DEFORMATION OBTAINED BY NON-LINEAR SEISMIC RESPONSE ANALYSIS OF R/C 9-STORY APARTMENT BUILDING WITH AND WITHOUT AIUF

Story		1	2	3	4	5	6	7	8	9
Building without AIUF										
Lateral seismic forces, kN		11601	11286	10589	9981	9548	9241	8803	7851	4723
Stages of defor-mation	In columns	Y	Y	Y	Y	Y	Y	Y	C	C
	In shear walls	U	U	U	U	U	Y	U	Y	Y
Building with AIUF										
Lateral seismic forces, kN		8332	8199	7154	6603	5130	4014	2927	1720	958
Stages of defor-mation	In columns	C	C	C	C	C	C	C	E	E
	In shear walls	Y	Y	Y	Y	C	C	C	C	E

E – elastic, C – cracking, Y– yielding, and U – ultimate stages of deformation

It was also decided to conduct dynamic tests of these buildings in two stages: first without AIUF, and then with it in resonance mode unprecedentedly used by its power vibration machine, which provided excitation of inertial horizontal loads allowing imitation of the design level seismic impact. The testing was held in transverse direction in resonance regime on the first vibration mode and it appeared necessary to make three stages of loading: the mass of eccentrics at the vibrator shafts was equal to 1920 kg at the 1st stage, 2880 kg at the 2nd stage and 3840 kg at the 3rd stage. Accordingly, the first mode vibration period increased gradually, growing up to 0.96 sec in the 3rd stage. In the building test without AIUF, the design intensity (VII by MSK-64 scale) impact was exceeded to about 6%, which was necessary, but insufficient condition for continuation of the experiment and permission of AIUF erection. It is needed to define how reliably the building would withstand the design impacts. In the given case the period at the 3rd testing stage appeared

to be greater than the initial one by 35%, while the design load was exceeded to 6%. Along with that no damage was observed in the bearing structures. This means the building is capable to withstand reliably the design intensity VII impact. That is why a decision was made to continue testing and permit erection of AIUF to upgrade the earthquake resistance of the building.

Before proceeding to the vibration testing of the building with AIUF, it was necessary to define its dynamic characteristics in order to tune the damper correctly. Its damping factor comprised 7.5%. Testing the building with AIUF again was held in resonance regime with masses of eccentrics at the vibrator shafts equal to those at the 3rd stage of testing without the AIUF, but in two vibration modes: AIUF and the building oscillate in the same phase (mode I/1), and AIUF oscillates in the anti-phase to the building (mode I/2). Comparison of the obtained shear forces at the ground floor level and displacements at the level of 9th floor slab have shown that in testing the building with AIUF only in the I/1 vibration mode thanks to the AIUF shear force and displacement are reduced by factors of 1.97 and 2.2, respectively. If the influence of I/2 vibration mode is considered as well, shear force will decline by a factor of 1.76. At the same time the drift of AIUF, or specifically the LRB displacement, exceeds the maximum drift of a story in the building by a factor of 4.3. However, this does not prevent using the AIUF space for various purposes, since its structures remain almost un-deformed. That is why AIUF compares favorably with AFUF.

Conclusions

The conducted experimental studies confirm that the efficiency of tuned mass dampers is rather high and that they are undoubtedly worth being employed to increase the seismic resistance of buildings. Tuned mass dampers in the form of AFUF or AIUF are suggested and presented. The efficiency of a single mass damper is not very high. Therefore, three dampers tuned to the first three vibrations modes of the building are considered much more effective as in this case optimal stiffness and mass correlations of dampers held the feature of significant reduction of shear forces and displacements (for about 2 times) compared to the building without TMD.

Deficiencies of AFUF are described and, thus the flexibility provided to the damper using LRBs is suggested. Transition from the concept of AFUF to the

concept of AIUF is justified. The non-linear seismic response analysis proves that with AIUF, acting as a TMD, seismic loads (the strain-stressed state level) experienced by the building could be reduced along the height of the building by about 2.5 times in average.

Dynamic testing of the existing 9-story building before and after erection of AIUF results in the conclusion that the proposed AIUF method leads to upgrading earthquake resistance of buildings and that AIUF brings to reduction of shear force at the ground floor level by a factor of 1.76 and at the same time the displacement at the 9th floor slab level decreases 2.2 times.

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